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Volume LIII, No. 7. LONDON, JULY, 1958.

EDITORIAL NOTES

The Results and Sources of Invention.

THE idea that science can quickly relieve mankind of most of the work now necessary is one that has been increasingly dwelt on in recent years, so much so, indeed, that a manual worker may be excused if he thinks that calls for more production are exhortations to scientists to discover substitutes that are more easily made and to invent machines that will work for us rather than pleas for people to work harder and for longer periods. Much recent propaganda relating to the possibilities of science has done a disservice to scientists when it suggests that they alone are able to cure the economic ills of the country. Such statements are misleading and are not likely to be borne out, for scientists are not magicians.

The foregoing sentences are taken from an Editorial Note in this journal of ten years ago, and it is interesting to see what has happened since they were written. Not only are there now more people at work, but they work as many hours during each week. In the 1930's there were seldom fewer than a million unemployed and about 20,000,000 at work. In January 1958 there were 200,000 unemployed and 23,000,000 at work. The average number of hours worked per week were 46·5 in the year 1938 and 46·6 in 1957. During the past twenty years there have been more scientific discovery, more invention, and more development of machinery and labour-saving devices than in any other similar period in history. It seems, however, that the more prolific the scientists and inventors the greater the amount of work there is to be done. The reason is, of course, that many goods that would formerly have been expensive luxuries have been brought within reach of more people. Longer holidays and often less work done during working time also call for more hours at a place of work to produce the same volume of goods, but generally it seems that people are willing to work longer in order to earn more money rather than to enjoy more leisure.

Another factor that has increased the demands of industry for labour is the ever-increasing number of people employed in discovering and developing new ideas and products. This subject is discussed in a recently-published book* in which the authors point out that most inventions cost very little. They are

* "The Sources of Invention." By John Jewkes, David Sawers, and Richard Stillerman. [London: Macmillan & Co., Ltd. Price 31s. 6d.]

generally either discovered by accident or as a by-product of research on a different subject, but their development is enormously costly. Among the figures given are the expenditure of £2,200,000 on nylon and £4,000,000 on terylene; these costs may be compared with £16,000 spent on the invention of Bessemer steel in the 1860's, and about £20,000 spent by Sir Frank Whittle and a few collaborators on the jet engine. What are described as development costs can, however, be very misleading. They may include only the work in the laboratory, or they may also include the cost of a pilot plant, a full-scale production plant, and sometimes the cost of advertising a new product. However this may be, the authors are critical of some of the approaches to research. Some firms spend money on what they call research merely because it is thought the proper thing to do although they have no specific end product or improvement in view. Others glorify their day-to-day routine testing by the word research, and include other expenditures under the same heading in order to give the impression that they are spending large sums on seeking and developing new processes or products. Still others describe as research the testing of products submitted to them; this is particularly the case in some Government research establishments; for example, in the Government establishments concerned with building and civil engineering the discovery of new processes or materials has been negligible. In a report of the Organisation for European Economic Co-operation the authors state that they saw laboratories that had been created only because the directors of a company "had been caught in a research boom and had established laboratories without any clear ideas of what the laboratories could do".

The discovery of new ideas does not depend only upon the number of people employed in a laboratory. The powers of discovery and invention are given to few and, although these men may need trained assistants, it is too often thought that if an expensive laboratory be filled with science graduates something useful is bound to be discovered. The authors of this book ask whether the concentration of so many research workers in large laboratories is an unmixed blessing in view of the fact that so many inventions have been produced by people who are not organised in this way: "Attempts to make research systematic do not guarantee results . . . The industrial laboratory does not appear to be a particularly favourable environment for invention." Sir Alexander Fleming, the discoverer of penicillin, has expressed the view that the employment of a team to undertake research is the worst possible way of starting, because "although a team will do some work it is not likely to initiate anything. . . . Organisation, attendance at committees, paper work, regular hours spent in a laboratory, are liable to quench originality." Certainly the large industrial research organisations have not been responsible for invention and discovery to anything like the extent that could be expected in relation to the numbers of people they employ and their cost.

Many of the most useful ideas have come from men unattached to an organisation and from the freer atmosphere of the universities. It may well be that much of the work done in industrial laboratories could be better done at the universities to the advantage of the nation and of the students, many of whom to-day are so busy filling their heads with what Dr. Glover called "loads of learned lumber" that they have no time to think; unfortunately too many graduates, in the words of Montaigne, "Know the theory of all things, but you must seek who shall put it into practice."

Restraint of Slabs by Edge-beams.

By L. S. MÜLLER.

THE restraint by marginal beams of slabs spanning in one direction may be determined approximately in certain cases if it is assumed that: (1) The bending of the columns supporting the beams is neglected. The deflection of the columns generally is not of a similar magnitude to that of the slabs unless the columns are exceptionally slender; therefore neglect of the deflection of the columns seldom greatly influences the restraining moments; in doubtful cases the values obtained by the approximate method may easily be corrected. (2) The edge-beam is continuous over several equal spans. (3) Internal supports of continuous slabs are assumed to be free supports.

Notation (*Figs.* 1 and 3).

L is the span of edge-beam; l, span of slab; B, width of beam; D, height of beam; $J = \beta B^3 D$, torsional stiffness of beam; d, thickness of slab; $K = \dfrac{d^3}{12l}$, stiffness of slab; K_u, stiffness of upper column; K_l, stiffness of lower column; E, Young's modulus for concrete; G, modulus of rigidity; $\mu = \frac{1}{6}$, Poisson's ratio for concrete; β, a constant depending upon ratio $\dfrac{D}{B}$; ϕ or ϕ', slope of slab at beam; ψ or ψ', twist of edge-beam per unit length; $w = g + p$, total load on slab; g, dead load; p, live load.

CASE 1(a), *Fig.* 3.—Single span supported on both sides by equal beams. The support moment of the slab elastically fixed is $M = 2EK\phi - \dfrac{wl^2}{12} = M_t$, that is the torsional moment of the edge-beam. The differential equation of torsion is

$$\frac{M_t}{GJ} = \frac{d^2\phi}{dx^2} = \frac{1}{GJ}\left(2EK\phi - \frac{wl^2}{12}\right) = \psi \qquad . \quad . \quad . \quad (1)$$

That is
$$\phi = \frac{1}{2EK}\left(GJ\psi + \frac{wl^2}{12}\right).$$

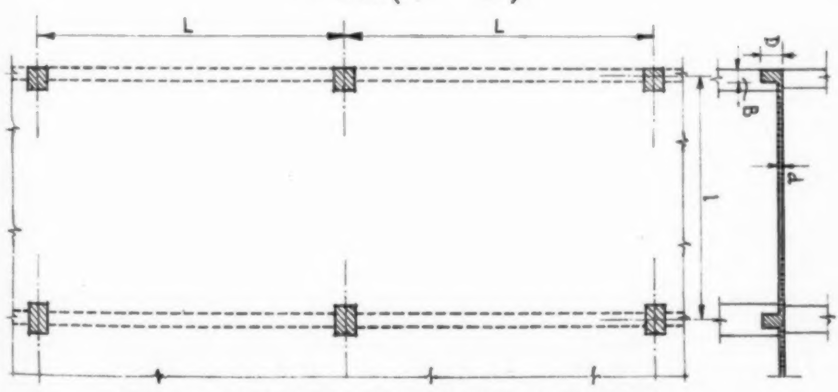
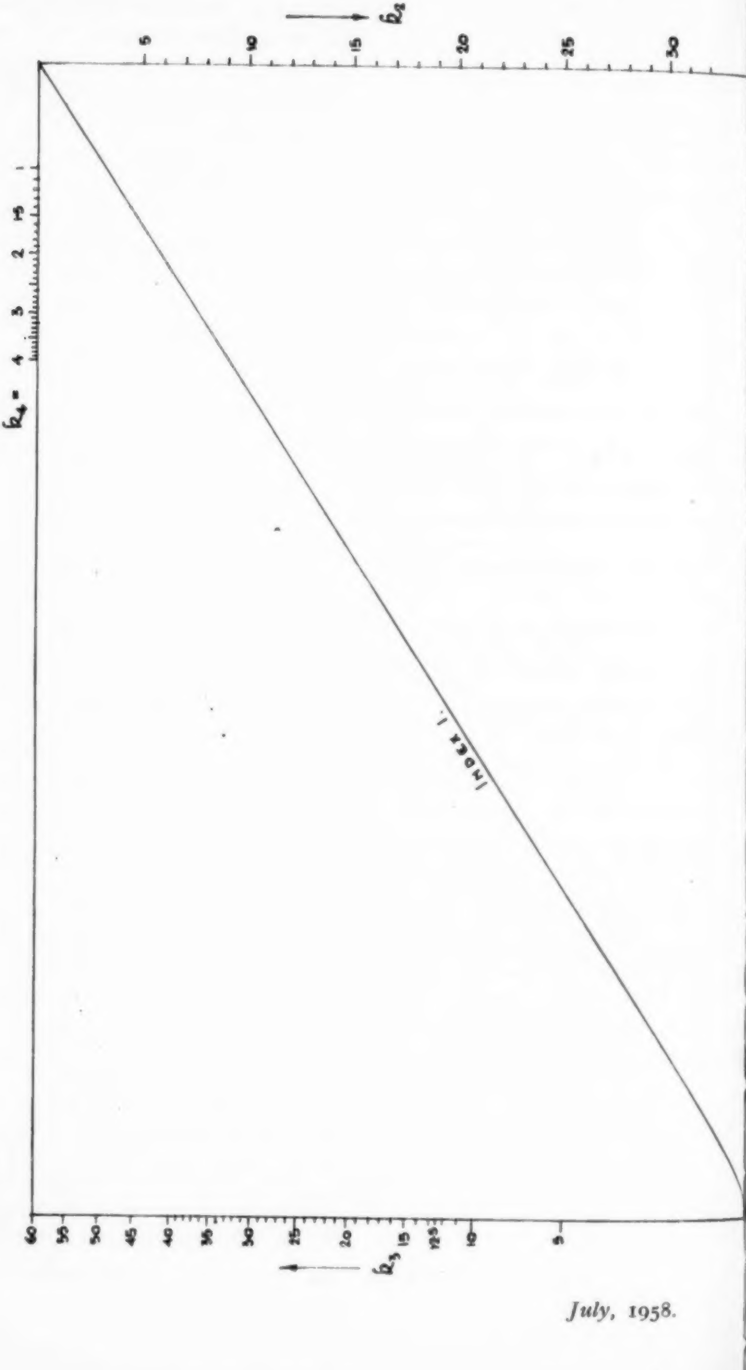


Fig. 1.—A Continuous Slab restrained by Edge-beams.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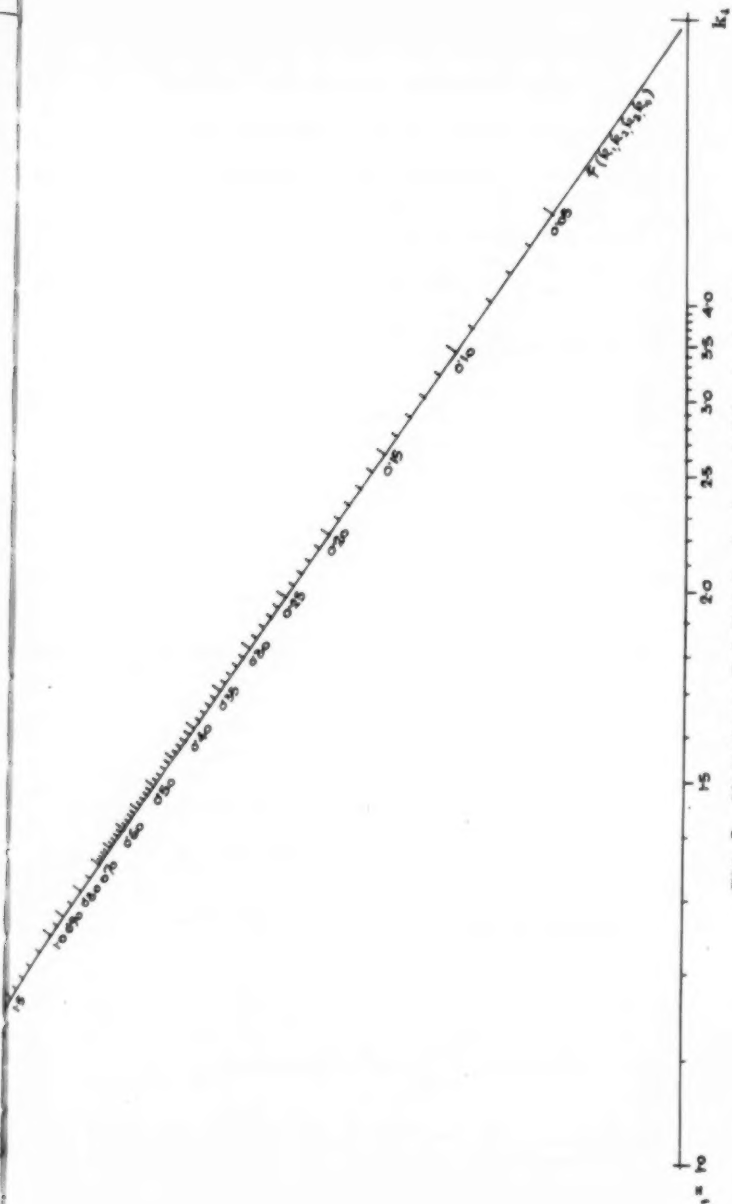


Fig. 2.—Nomogram Relating k_1, k_2, k_3 and $F(k_1, k_2, k_3, k_4)$.

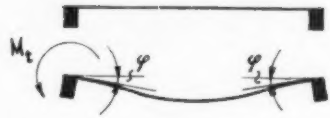


Fig. 3.—Slab Spanning between Two Similar Beams.

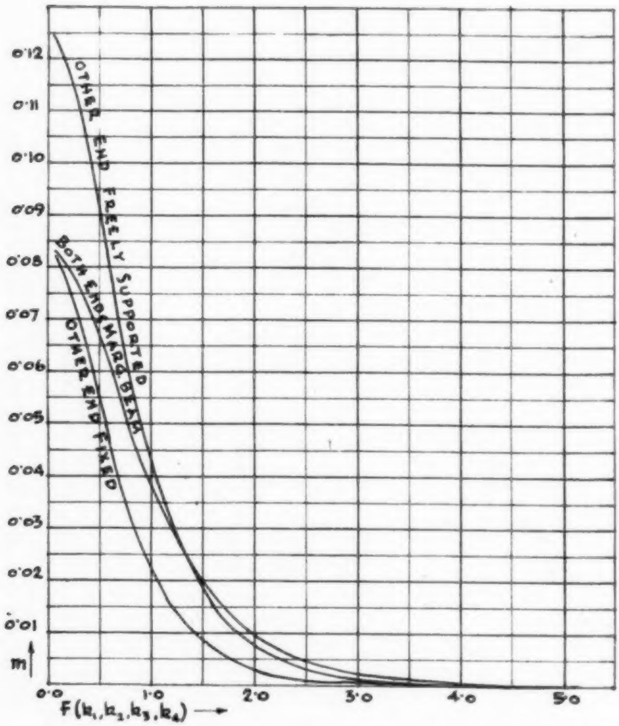


Fig. 4.—Bending-moment Factors for a Single Span.

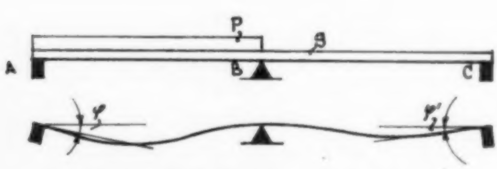


Fig. 5.—Slab of Two Equal Spans.

In order to obtain a new relationship, differentiate this equation twice, that is

$$\frac{d\phi}{dx} = \frac{GJ}{2EK}\cdot\frac{d\psi}{dx}; \quad \frac{d^2\phi}{dx^2} = \frac{GJ}{2EK}\cdot\frac{d^2\psi}{dx^2} = \psi; \quad \text{or} \quad \frac{d^2\psi}{dx^2} - \frac{2EK}{GJ}\psi = 0.$$

Denoting $\frac{2EK}{GJ}$ by α^2,

$$\psi = C_1 e^{\alpha x} + C_2 e^{-\alpha x} \quad . \quad . \quad . \quad . \quad (2)$$

Assuming that the origin of the co-ordinate system is at midspan of the edge-beam, the values of ψ at $x = +\frac{L}{2}$ and $x = -\frac{L}{2}$ are equal:

$$\psi = C_1 e^{\alpha\frac{L}{2}} + C_2 e^{-\alpha\frac{L}{2}} = C_1 e^{-\alpha\frac{L}{2}} + C_2 e^{\alpha\frac{L}{2}}.$$

That is

$$C_1 = C_2 = \frac{C}{2} \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Therefore

$$\psi = C\frac{e^{\alpha x} + e^{-\alpha x}}{2} = C\cosh\alpha x \quad . \quad . \quad . \quad . \quad (4)$$

From the initial assumption that $\phi = 0$ where $x = \pm\frac{L}{2}$, and from (1) and (4),

$$C\cosh\alpha\frac{L}{2} = -\frac{1}{GJ}\cdot\frac{wl^2}{12}, \quad \text{or} \quad C = -\frac{1}{GJ\cosh\alpha\frac{L}{2}}\cdot\frac{wl^2}{12} \quad . \quad . \quad (5)$$

Therefore $M_t = GJ\psi = -\frac{\cosh\alpha x}{\cosh\alpha\frac{L}{2}}\cdot\frac{wl^2}{12}$, and at midspan, where $x = 0$, $\cosh\alpha x = 1$,

and the least moment of restraint for the slabs is

$$M_t min. = -\frac{wl^2}{12\cosh\alpha\frac{L}{2}} \quad . \quad . \quad . \quad . \quad (6)$$

To evaluate the moment factors, write $\alpha = \sqrt{2}\sqrt{\frac{EK}{GJ}}$.

$G = \frac{E}{2(1+\mu)}$; writing $\mu = \frac{1}{6}$, $G = \frac{3}{7}E$. Also $K = \frac{d^3}{12l}$, $J = \beta B^3 D$.

Therefore

$$\alpha\frac{L}{2} = \sqrt{2}\frac{L}{2}\sqrt{\frac{EK}{GJ}} = \sqrt{2}\frac{L}{2}\sqrt{\frac{E\frac{d^3}{12l}}{\frac{3}{7}E\beta\frac{D}{B}B^4}} = \sqrt{2}\frac{0\cdot22}{\sqrt{\beta}}\cdot\frac{d}{B}\cdot\frac{L}{B}\cdot\frac{1}{\sqrt{\frac{D}{B}}\sqrt{\frac{l}{d}}} \quad . \quad (7)$$

Writing $\frac{B}{d} = k_1$, $\frac{L}{B} = k_2$, $\frac{l}{d} = k_3$, $\frac{D}{B} = k_4$, then

$$\alpha\frac{L}{2} = \sqrt{2}\frac{0\cdot22}{\sqrt{\beta}}\cdot\frac{k_2}{k_1}\cdot\frac{1}{\sqrt{k_3 k_4}} = \sqrt{2}F(k_1, k_2, k_3, k_4) \quad . \quad . \quad (8)$$

The values of $F(k_1, k_2, k_3, k_4)$ may be read from the nomogram (*Fig.* 2). As the argument of $\cosh \alpha\frac{L}{2}$ differs from $F(k_1, k_2, k_3, k_4)$ by only a factor (in this case $\sqrt{2}$), $F(k_1, k_2, k_3, k_4)$ is used as a parameter in the diagram of bending-moment factors $\dfrac{1}{12 \cosh \alpha\frac{L}{2}}$ in *Fig.* 4.

For the construction of the nomogram, values of β are taken from Timoshenko's "Theory of Elasticity", but, in order to obtain a continuous function between the boundaries $k_4 = 1$ and $k_4 = 4$, Timoshenko's approximate formula has been replaced by the empirical one

$$\beta = 0{\cdot}33811 - \frac{0{\cdot}24253}{0{\cdot}228 + k_4},$$

which is used for all the tabulated values. Thereby the scale of k_4 is actually the scale of $\sqrt{\beta k_4}$.

CASE 1(b).—A single span supported on one side by a beam and fixed on the opposite side. The restraining moment at the elastic support in this case is

$$M_t = 4EK\phi - \frac{wl^2}{12} \quad . \quad . \quad . \quad . \quad . \quad (9)$$

Therefore

$$\frac{M_t}{GJ} = \frac{d^2\phi}{dx^2} = \frac{1}{GJ}\left(4EK\phi - \frac{wl^2}{12}\right) = \psi \quad . \quad . \quad . \quad (10)$$

By reasoning similar to that in 1(a),

$$\psi = -\frac{1}{GJ}\,\frac{\cosh \alpha x}{\cosh \alpha\frac{L}{2}}\cdot\frac{wl^2}{12} \quad . \quad . \quad . \quad . \quad (11)$$

but in this case

$$\alpha = \sqrt{\frac{4EK}{GJ}} = 2\sqrt{\frac{EK}{GJ}} \quad . \quad . \quad . \quad . \quad (12)$$

and

$$M_t min. = -\frac{1}{\cosh \alpha\frac{L}{2}}\cdot\frac{wl^2}{12} \quad . \quad . \quad . \quad . \quad (13)$$

In this case $\cosh \alpha\frac{L}{2} = \cosh 2F(k_1, k_2, k_3, k_4)$. The moment factors

$$\frac{1}{12 \cosh \alpha\frac{L}{2}}$$

are given by the lower curve of *Fig.* 4.

CASE 1(c).—A single span supported on one side by a beam and freely supported on the opposite side. The moment at the elastic support is

$$M_t = 3EK\phi - \frac{wl^2}{8} \qquad (14)$$

$$\frac{M_t}{GJ} = \frac{d^2\phi}{dx^2} = \frac{1}{GJ}\left(3EK\phi - \frac{wl^2}{8}\right) = \psi \qquad (15)$$

Similarly, the solution is

$$\psi = -\frac{1}{GJ}\,\frac{\cosh \alpha x}{\cosh \alpha\frac{L}{2}}\cdot\frac{wl^2}{8} \qquad (16)$$

In this case

$$\alpha = \sqrt{3}\sqrt{\frac{EK}{GJ}} \qquad (17)$$

and

$$M_t min. = -\frac{1}{\cosh \alpha\frac{L}{2}}\cdot\frac{wl^3}{8} \qquad (18)$$

In this case $\cosh \alpha\frac{L}{2} = \cosh \sqrt{3}F(k_1, k_2, k_3, k_4)$.

The moment factors $\dfrac{1}{8 \cosh \alpha\frac{L}{2}}$ are given by the upper curve of *Fig.* 4.

CASE 2.—Two equal spans supported externally by two equal beams; one span is loaded with the dead and live loads and the other span is unloaded. With the notation of *Fig.* 5,

$$\left.\begin{aligned} M_A = M_t &= -\frac{5wl^2}{48} + \frac{gl^2}{48} + \frac{7EK\phi}{2} - \frac{EK\phi'}{2} \\ M_C = M_t' &= \frac{5gl^2}{48} - \frac{wl^2}{48} + \frac{7EK\phi'}{2} - \frac{EK\phi}{2} \end{aligned}\right\} \qquad (19)$$

$$\left.\begin{aligned} \frac{M_t}{GJ} = \frac{d^2\phi}{dx^2} &= \frac{1}{GJ}\left(\frac{7EK\phi}{2} - \frac{EK\phi'}{2} - \frac{5wl^2}{48} + \frac{gl^2}{48}\right) = \psi \\ \frac{M_t'}{GJ} = \frac{d^2\phi'}{dx^2} &= \frac{1}{GJ}\left(\frac{7EK\phi'}{2} - \frac{EK\phi}{2} + \frac{5gl^2}{48} - \frac{wl^2}{48}\right) = \psi' \end{aligned}\right\}. \qquad (20)$$

Solving for ϕ and ϕ',

$$\left.\begin{aligned} \phi &= \frac{\phi'}{7} + \frac{1}{3{\cdot}5EK}\left(GJ\psi + \frac{5wl^2}{48} - \frac{gl^2}{48}\right) \\ \phi' &= \frac{\phi}{7} + \frac{1}{3{\cdot}5EK}\left(GJ\psi' - \frac{5gl^2}{48} + \frac{wl^2}{48}\right) \end{aligned}\right\} \qquad (21)$$

After differentiating twice simultaneous differential equations (22) are obtained.

$$\psi - \frac{\psi'}{7} = \frac{GJ}{3{\cdot}5EK}\cdot\frac{d^2\psi}{dx^2}; \quad \psi' - \frac{\psi}{7} = \frac{GJ}{3{\cdot}5EK}\cdot\frac{d^2\psi'}{dx^2}. \qquad (22)$$

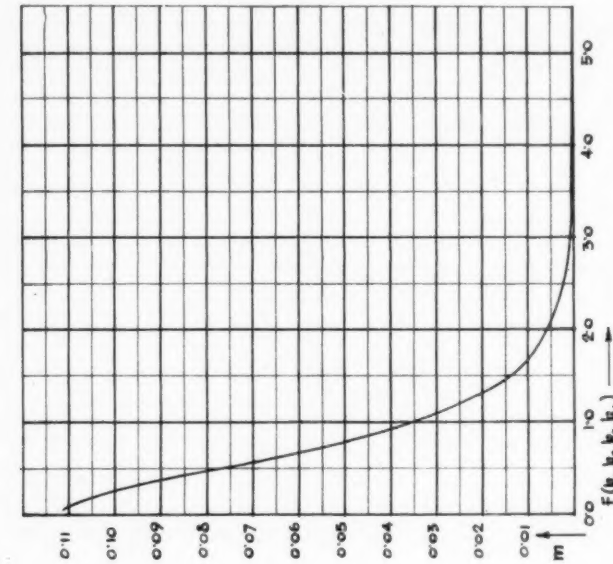


Fig. 7.—Bending-moment Factors for Three Equal Spans.

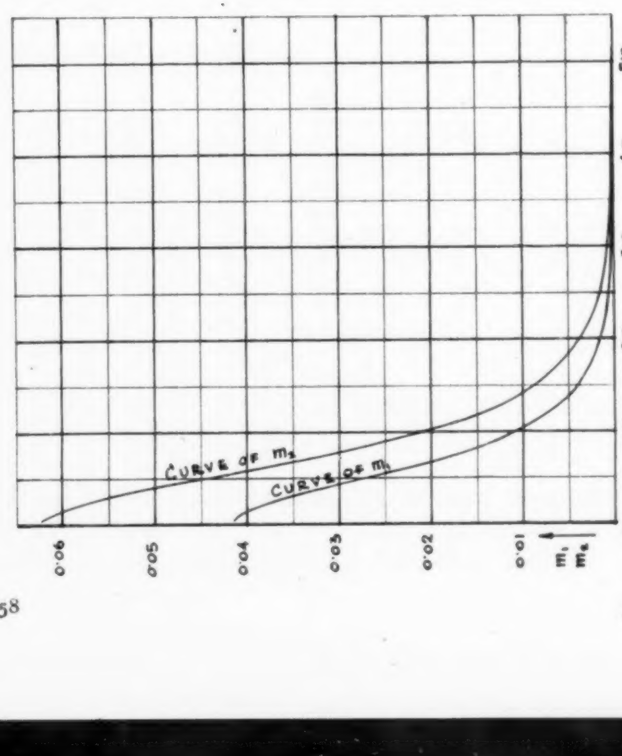


Fig. 6.—Bending-moment Factors for Two Equal Spans.

Writing α for $\sqrt{3\cdot5}\sqrt{\dfrac{EK}{GJ}}$, (22) may be reduced to one differential equation of a higher order :

$$\frac{d^4\psi}{dx^4} - 2\alpha^2\frac{d^2\psi}{dx^2} + \frac{48}{49}\alpha^4\psi = 0 \quad . \quad . \quad . \quad . \quad (23)$$

The characteristic equation is

$$\lambda^4 - 2\alpha^2\lambda^2 + \frac{48}{49}\alpha^4 = 0 \quad . \quad . \quad . \quad . \quad (24)$$

and the four roots are

$$\lambda_1 = +\,1\cdot06905\alpha,\ \lambda_2 = -\,1\cdot06905\alpha,\ \lambda_3 = +\,0\cdot92582\alpha,\ \lambda_4 = -\,0\cdot92582\alpha.$$

The general solution is therefore

$$\psi = C_1e^{1\cdot06905\alpha x} + C_2e^{-1\cdot06905\alpha x} + C_3e^{0\cdot92582\alpha x} + C_4e^{-0\cdot92582\alpha x} \quad . \quad (25)$$

Again taking the origin at midspan of the beam, from the condition that

$$(\psi)_{+\frac{L}{2}} = (\psi)_{-\frac{L}{2}},\ C_1 = C_2 = \frac{A}{2},\ C_3 = C_4 = \frac{B}{2},$$

$$\psi = A\cosh 1\cdot06905\alpha x + B\cosh 0\cdot92582\alpha x \quad . \quad . \quad . \quad (26)$$

Solving (22) for ψ',

$$\psi' = -\,A\cosh 1\cdot06905\alpha x + B\cosh 0\cdot92582\alpha x \quad . \quad . \quad (27)$$

From the condition that $\phi = \phi' = 0$, where $x = \dfrac{L}{2}$,

$$\left.\begin{aligned} A &= -\frac{1}{GJ\cosh 1\cdot06905\alpha\frac{L}{2}}\cdot\frac{(w+g)l^2}{24} \\ B &= -\frac{1}{GJ\cosh 0\cdot92582\alpha\frac{L}{2}}\cdot\frac{(w-g)l^2}{16} \end{aligned}\right\} \quad . \quad . \quad . \quad (28)$$

Substituting this in equation (26),

$$\psi = -\frac{\cosh 1\cdot06905\alpha x}{GJ\cosh 1\cdot06905\alpha\frac{L}{2}}\cdot\frac{(w+g)l^2}{24} - \frac{\cosh 0\cdot92582\alpha x}{GJ\cosh 0\cdot92582\alpha\frac{L}{2}}\cdot\frac{(w-g)l^2}{16} \quad . \quad (29)$$

and

$$M_t min. = GJ(\psi)_{x=0}$$

$$= -\frac{1}{24\cosh 1\cdot06905\alpha\frac{L}{2}}\cdot(w+g)l^2 - \frac{1}{16\cosh 0\cdot92582\alpha\frac{L}{2}}(w-g)l^2 \quad . \quad (30)$$

As $1\cdot06905\sqrt{3\cdot5} = 2$ and $0\cdot92582\sqrt{3\cdot5} = \sqrt{3}$ the partial moment factors m_1 of $(w+g)l^2$ are half of those for Case 1(b), and the partial moment factors m_2 of $(w-g)l^2$ are half of those for Case 1(c). These factors are given in *Fig.* 6.

CASE 3.—Three equal spans supported externally by two equal beams ; outer spans loaded and inner span unloaded.

The moment at the elastic support is

$$M_t = \frac{1}{3}\left[10EK\phi - \left(w - \frac{g}{4}\right)\frac{l^2}{3}\right] \quad . \quad . \quad . \quad . \quad (31)$$

$$\frac{M_t}{GJ} = \frac{d^2\phi}{dx^2} = \frac{1}{3GJ}\left[10EK\phi - \left(w - \frac{g}{4}\right)\frac{l^2}{3}\right] = \psi \quad . \quad . \quad (32)$$

and

$$\phi = \frac{1}{10EK}\left[3GJ\psi + \left(w - \frac{g}{4}\right)\frac{l^2}{3}\right].$$

After two differentiations, $\dfrac{d^2\psi}{dx^2} - \dfrac{10EK}{3GJ}\psi = 0$. Denoting $\sqrt{\dfrac{10}{3}}\sqrt{\dfrac{EK}{GJ}}$ by α,

$$\psi = C \cosh \alpha x \quad . \quad . \quad . \quad . \quad (33)$$

Where $x = \dfrac{L}{2}$, $\phi = 0$ and, from (32) and (33),

$$C = -\frac{w - \dfrac{g}{4}}{9GJ\cosh \alpha\dfrac{L}{2}} \quad . \quad . \quad . \quad . \quad . \quad (34)$$

and

$$\psi = -\frac{\cosh \alpha x}{GJ\cosh \alpha\dfrac{L}{2}} \cdot \frac{\left(w - \dfrac{g}{4}\right)l^2}{9} \quad . \quad . \quad . \quad . \quad (35)$$

Where $x = 0$, $\cosh \alpha x = 1$; and

$$M_t min. = -\frac{1}{9\cosh \alpha\dfrac{L}{2}} \cdot \left(w - \frac{g}{4}\right)l^2. \quad . \quad . \quad . \quad (36)$$

The moment factors $\dfrac{1}{9\cosh \alpha\dfrac{L}{2}}$ are given in *Fig.* 7.

Examples.

EXAMPLE 1.—A continuous slab of three equal spans supports a load g of 75 lb. per square foot and $p = 40$ lb. per square foot. The dimensions are d, 4 in.; B, 1 ft. 4 in.; D, 3 ft. 4 in.; l, 12 ft. 1 in.; L, 20 ft.; k_1, $\frac{16}{4} = 4$; k_2, $\frac{240}{16} = 15$; k_3, $\frac{145}{4} = 36{\cdot}25$; k_4, $\frac{40}{16} = 2{\cdot}5$. From *Fig.* 2,

$$F(k_1, k_2, k_3, k_4) = 0{\cdot}174,$$

and, from *Fig.* 7, $m = 0{\cdot}106$. Therefore

$$M_t min. = 0{\cdot}106(115 - \tfrac{75}{4})12{\cdot}08^2 = 1485 \text{ ft.-lb.}$$

In an extreme case of a heavy slab and an edge-beam supported by comparatively slender columns it may be advisable to examine the influence of the deflection of the column upon the restraining moment of the beam. This may

be done approximately as follows. The moment transferred from beams on both sides of a column in Case 1(a), for example, is

$$\bar{M} = 2\int_0^{\frac{L}{2}} M_t\,dx = -\frac{2wl^2}{12}\cdot\frac{1}{\cosh \alpha\frac{L}{2}}\int_0^{\frac{L}{2}} \cosh \alpha x = -\frac{wl^2}{6\alpha}\tanh \alpha\frac{L}{2} \quad . \quad (37)$$

The slope of the column at the connection with the beam, assuming that the columns are fixed at the remote ends, is

$$\phi_C = \frac{\bar{M}}{4E(K_u + K_l)}.$$

In equation (1), if $x = \pm\frac{L}{2}$, $\phi \neq 0$, but $\phi = \phi_C$. Therefore

$$C \cosh \alpha\frac{L}{2} = \frac{1}{GJ}\left[\frac{2EK}{4E(K_u + K_l)}\cdot\frac{wl^2 \tanh \alpha\frac{L}{2}}{6\alpha} - \frac{wl^2}{12}\right]$$

$$= C \cosh \alpha\frac{L}{2} = \frac{1}{GJ}\cdot\frac{wl^2}{12}\left(\frac{K}{K_u + K_l}\cdot\frac{\tanh \alpha\frac{L}{2}}{\alpha} - 1\right) \quad . \quad . \quad (38)$$

$$C = \frac{1}{GJ \cosh \alpha\frac{L}{2}}\cdot\frac{wl^2}{12}\left(\frac{K}{K_u + K_l}\cdot\frac{\tanh \alpha\frac{L}{2}}{\alpha} - 1\right) \quad . \quad . \quad (39)$$

and

$$M_t min. = -\frac{wl^2}{12 \cosh \alpha\frac{L}{2}}\left(1 - \frac{K}{K_u + K_l}\cdot\frac{\tanh \alpha\frac{L}{2}}{\alpha}\right) \quad . \quad . \quad (40)$$

EXAMPLE 2.—A slab is supported on both sides by beams. The dimensions are d, 4 in.; B, 10 in.; L, 16 ft. 8 in.; l, 20 ft.; D, 2 ft. 1 in. The upper column is 10 in. by 10 in. in cross section and the lower column 10 in. by 12 in. The height of both columns is 10 ft. 10 in.

$$k_1 = \frac{10}{4} = 2{\cdot}5,\ k_2 = \frac{200}{10} = 20,\ k_3 = \frac{240}{4} = 60,\ k_4 = \frac{25}{10} = 2{\cdot}5.$$

$$K = \frac{4^3}{12 \times 240} = 0{\cdot}0222,\ K_u = \frac{10^4}{12 \times 130} = 6{\cdot}4,\ K_l = \frac{10 \times 12^3}{12 \times 130} = 11{\cdot}05.$$

From *Fig.* 2, $F(k_1, k_2, k_3, k_4) = 0{\cdot}29$, and from *Fig.* 4 $m = \frac{1}{12 \cosh \alpha\frac{L}{2}} = 0{\cdot}0769.$

Therefore $\cosh \alpha\frac{L}{2} = \frac{1}{12 \times 0{\cdot}0769} = 1{\cdot}0836.$

Hence $\alpha\frac{L}{2} = 0{\cdot}40726$ and $\alpha = \frac{0{\cdot}40726}{100} = 0{\cdot}0040726$; therefore $\tanh \alpha\frac{L}{2} = 0{\cdot}38614$.

The reduction factor is $\left(1 - \frac{0{\cdot}0222}{6{\cdot}4 + 11{\cdot}05}.\frac{0{\cdot}38614}{0{\cdot}0040726}\right) = 1 - 0{\cdot}12 = 0{\cdot}88$, and the moment factor $0{\cdot}0769 \times 0{\cdot}88 = 0{\cdot}0678$.

GENERAL CASE.—The slab has different spans and different loads. The support moment M_2 of the first internal support (*Fig.* 8) opposite the beam is

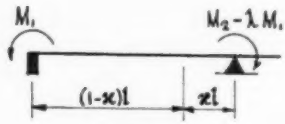


Fig. 8.

known, assuming free support throughout. Also the distance of the fixed-point near the same support $\varkappa l$ should be known, assuming complete fixity at the external support. If the restraining moment of the edge-beam is M_1, M_2 will be reduced by almost λM_1 where $\lambda = \frac{\varkappa}{1 - \varkappa}$. According to Mohr's theorem, if the full load on the external span is w,

$$\phi_1 = -\frac{1}{6EK}(2M_1 + M_2 - \lambda M_1) + \frac{1}{EK}.\frac{wl^2}{24} \quad . \quad . \quad . \quad (41)$$

$$M_1 = -\frac{6}{2-\lambda}EK\phi_1 - \frac{M_2}{2-\lambda} + \frac{wl^2}{4(2-\lambda)} = M_t.$$

$$\frac{M_t}{GJ} = \frac{d^2\phi}{dx^2} = \frac{1}{GJ}\left[\frac{6}{2-\lambda}EK\phi + \frac{M_2}{2-\lambda} - \frac{wl^2}{4(2-\lambda)}\right] = \psi \ . \quad (42)$$

$$\phi = \frac{2-\lambda}{6EK}\left[GJ\psi - \frac{M_2}{2-\lambda} + \frac{wl^2}{4(2-\lambda)}\right] \quad . \quad . \quad . \quad (43)$$

By differentiating twice $\frac{d^2\psi}{dx^2} - \frac{6EK}{(2-\lambda)GJ}.\psi = 0$, and with $\frac{6EK}{(2-\lambda)GJ} = \alpha^2$ the solution is

$$\psi = \frac{M_2 - 0{\cdot}25wl^2}{(2-\lambda)GJ\cosh\alpha\frac{L}{2}}.\cosh\alpha x \ . \quad . \quad . \quad . \quad (44)$$

and $M_t min. = \dfrac{M_2 - 0{\cdot}25wl^2}{(2-\lambda)\cosh\alpha\frac{L}{2}}$.

Write $\alpha\frac{L}{2}$ in the form $\sqrt{\frac{6}{2-\lambda}}.\frac{L}{2}\sqrt{\frac{EK}{GJ}} = \sqrt{\frac{6}{2-\lambda}}F(k_1, k_2, k_3, k_4)$ and use the nomogram.

It should be emphasised that whenever the restraint of edge-beams is taken into account in the dimensions of the slab, adequate reinforcement to resist torsion should be provided in the beams. The torsional moment may be calculated from equation (37), or from the similar equations in other cases.

BIBLIOGRAPHY.

(1) "Degree of Fixity of Reinforced Concrete Slabs at the Marginal Beams", by A. Shmelkes. (Journal of the Association of Engineers and Architects in Palestine. In Hebrew. Tel-Aviv, November 1944.) (2) "Die Einspannung einer rechteckigen Deckenplatte in einem elastisch verdrillbaren Randträger", by M. Klasmer. (Author's own publication. Jerusalem, 1946.) (3) "Über die Berücksichtigung der Torsionssteifigkeit der Randbalken von Stahlbetondecken", by W. Sager. (Beton und Stahlbetonbau, Vol. 45, No. 10. Berlin, 1950.) (4) For beams and slabs with elastically-restrained end-supports see "Stresses in Framed Structures" (Appendix A), by Hool and Kinne.

Book Reviews.

"Handbook of Noise Control." Edited by Cyril M. Harris. (London: McGraw-Hill Publishing Co., Ltd. Price £6 4s.)

FOLLOWING several chapters on the properties of noise and the effect of noise on human beings, there are chapters on the measurement and isolation of noise, acoustical materials, transmission of sound through materials, and the requirements relating to the prevention of noise of some U.S.A. building codes. The editor is an associate professor of electrical engineering at Columbia University, and the chapters have been contributed by a number of U.S.A. writers on the subject.

"Prestressed Concrete." By P. B. Morice and E. H. Cooley. (London: Sir Isaac Pitman & Sons, Ltd. Price 57s. 6d.)

THIS book of 394 pages provides information on the theory, design, and use of prestressed concrete. It will undoubtedly be of use to students and to practising engineers, but its usefulness could have been increased. For example, 73 pages are devoted to descriptions of various systems of prestressing; this is much more than is required to meet the needs of students, and most designers will prefer to use the data issued by the owners of the systems.

The discussion of losses of prestress could usefully be expanded, and the statement on page 43 that the losses are "generally taken to be asymptotic to a value of 84 per cent. of the initial prestressing force" needs to be corrected. The treatment of ultimate-load design is brief; the statement in the preface that "an elastic theory is now generally recognised as adequate" is not entirely correct and the desirability of checking all designs for conditions at ultimate-load is not sufficiently stressed.

There is an interesting chapter on the design of tanks and pipes, and the discussion of design considerations is helpful. Some of the contents of the book have been issued by the Cement and Concrete Association in the form of brochures.

"International Business Register, 1958." (London: Bottin International Co. Price £4 15s.)

IT is perhaps too much to expect that a directory of manufacturers and wholesalers throughout the world of products ranging from abbatoir equipment to zip fasteners should be complete and accurate, even within the scope of more than 2000 pages. Some of the sections we have checked leave much to be desired in these respects. There are also pages relating to many countries with maps and information on the industries, customs duties, consulates, chambers of commerce, and lists of names and addresses of businesses.

"Elementare Schalenstatik." By A. Pflüger. (Berlin: Springer-Verlag. Price 19.50 D.M.)

IN this second and enlarged edition the fundamental theory of thin shell design is developed. Formulæ are given including various cases of spherical, conical and cylindrical shells and barrel vaults. The book is primarily intended for students and does not give any particular solutions leading to quick and accurate results.

"Planungsgrundlagen für den Städtebaulichen Ideenwettbewerb Hauptstadt Berlin." (Berlin: Wilhelm Ernst & Sohn. Price 39 D.M.)

IN this volume are given plans, printed to a large scale and in colour, of the proposed rebuilding of the city of Berlin.

Three-story School with Precast Walls.

A SCHOOL at Evreux, France, is being built entirely of precast concrete; it will accommodate 1600 students, 800 of whom will be boarders. The four main class-room buildings (*Fig.* 1) are each of four stories 225 ft. long by 40 ft. wide. In addition there will be an administrative building with rooms for special classes, a gymnasium, living accommodation for the boarders, and kitchens and laundry.

The walls comprise precast slabs 13 ft. high by 5 ft. 9 in. wide, in many of which metal window frames are fixed. The slabs have a thin outer layer of normal concrete behind which is a filling of insulating concrete which is retained by a thin layer of concrete made with pozzolanic aggregate and sand. The vertical edges of the slabs are recessed so that columns may be cast in place between adjoining slabs without the use of shutters (*Fig.* 2). The slabs are made by machine. The floors consist of precast beams at 5 ft. 9 in. centres spanning between the walls and supporting precast slabs. Ties may be formed by placing concrete in the tops of the beams, which are grooved.

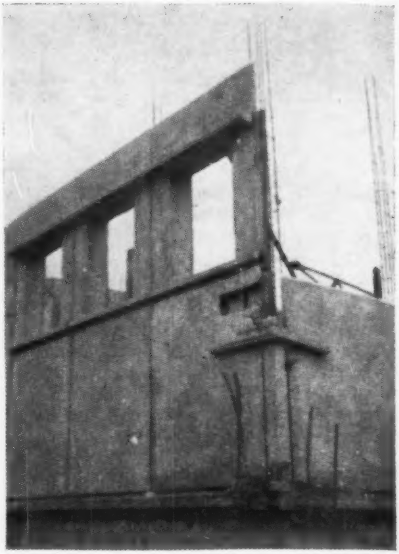


Fig. 2.—A Partly-completed Wall.

The buildings are described in the Belgian journal "La Technique des Travaux" for May–June, 1957. They were designed by M. Pierre Bailleau, Architecte en Chef des Bâtiments Civils et Palais Nationaux, and built by the Société des Constructions Edmond Coignet who also devised the method of construction.

Fig. 1.—A Class-room Building.

Reconstruction of Two Bridges in Fife.

PREFABRICATED BEAM REINFORCEMENT AND SOFFITS.

THE bridges shown in *Fig.* 1 have recently been built to replace a stone arch bridge of three spans over the River Leven and a steel parapet-girder bridge over the adjacent railway.

The new bridge over the river consists of a filled-spandrel barrel arch with a clear span of 135 ft., supported by plain concrete abutments which are in turn supported by precast concrete piles.

In the case of the bridge over the railway, the reinforcement for the beams was assembled and welded together and the lower part embedded in concrete, which forms the soffit of the beam; *Fig.* 2 shows the reinforcement and the soffit ready for transporting to the site, and they are seen in position in *Fig.* 3.

Precast slabs supported on the precast part of the beam (*Fig.* 3) form the soffits of the service ducts. Additional precast slabs placed at a higher level support the deck, which was cast in place when the upper parts of the beams were concreted. The prefabricated reinforcement was placed in position by means of a gantry spanning the railway and travelling on tracks on the tops of the abutments; this consisted of Bailey bridge sections with carriages travelling on rails

Fig. 1.

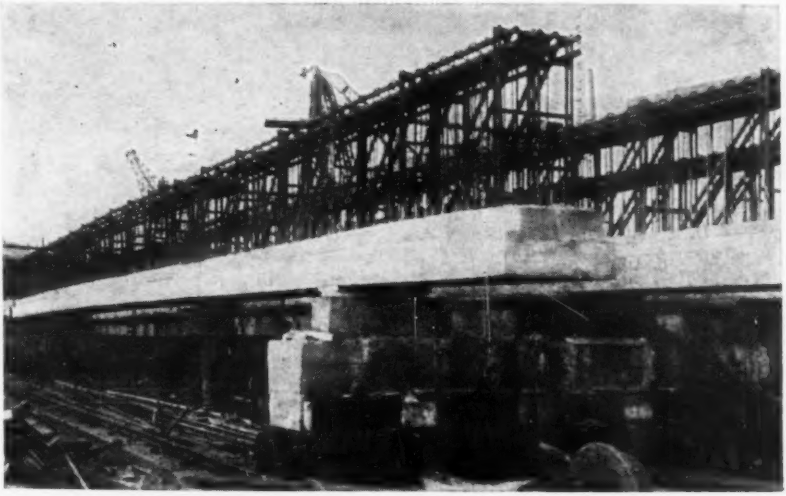


Fig. 2.—Prefabricated Reinforcement and Soffits.

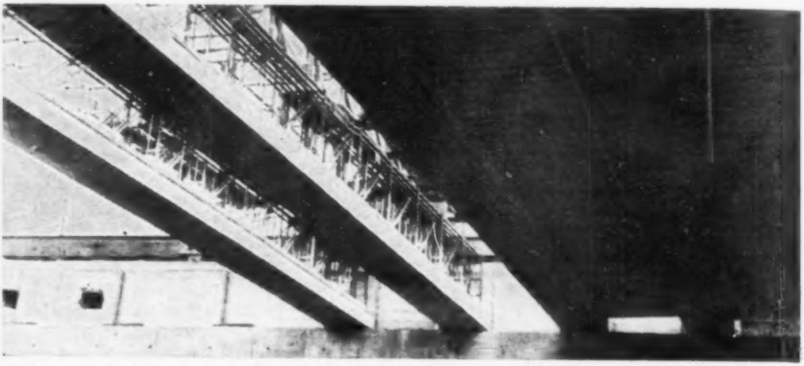


Fig. 3.—Left : Prefabricated Reinforcement and Soffits. Right : Slabs in Position on Soffits.

Fig. 4.—Gantry used for Placing Reinforcement.

at both ends (*Fig.* 4). By this means the work of reconstruction caused little interference with the sidings under the bridge. The retaining walls and parapets are faced with sandstone.

The County Surveyor is Mr. T. McCallum. The work was constructed to designs prepared by Messrs. F. A. Macdonald & Partners, who also supervised the construction. The main contractors were Messrs. Melville, Dundas, & Whitson, Ltd.

A Church at Glasgow.

St. Stephen's Roman Catholic church at Dalmuir, Glasgow, now nearing completion, replaces a church which was totally destroyed during an air-raid in 1942. It is a reinforced concrete structure with brick walls and is arranged as shown in *Fig.* 2. The entrance hall is flanked by vestibules, one of which leads to the baptistery and the other to the creche. A cloakroom, and reinforced concrete stairs leading to the gallery, adjoin the entrance hall.

The gallery is a 9-in. slab cantilevered at one side to form a canopy over the main entrance. The slabs over the cloakroom and adjoining the staircase are extended to form canopies over the secondary entrances. On one side of the gallery is a passage leading to the belfry tower, which is over the baptistery. The ground floor in this part of the church consists of two slabs 3 in. thick with a layer of asphalt between them, and is finished with precast concrete tiles laid on mortar. The pitched roof is of timber covered with tiles on battens.

The nave has aisles on each side, and the sanctuary and high altar are at one end of the church. The nave is about 120 ft. long and 58 ft. wide, and the transept is 80 ft. wide. The ground floor is similar to that in the entrance hall, and reinforced concrete ducts for heating purposes are provided. The floor is finished with wooden blocks ⅞ in. thick laid on mortar.

The main structure consists of eleven gable frames, of 36-ft. span by 52 ft. high, at 14-ft. centres (*Fig.* 3). Over the transept there are two cross-frames of 50 ft. span and 52 ft. high (*Fig.* 1). The main frames are 21 in. by 12 in. in cross section at the ground floor and increase to 42 in. by 12 in. at the eaves, from which they taper to 21 in. by 12 in. at the apex. The frames are pin-jointed at the base, and resist the loads from the roof, the 11-in. cavity walls, the flat roofs of the aisles, and the wind pressure.

Fig. 1.

The cross-frames over the transept are fully fixed at the apex and pin-jointed at the bases; each base supports the three frames meeting at each corner. All the bases are square, and the pressure on the ground is about 2 tons per square foot.

The roof has a slope of 35 deg. and consists of precast L-shape purlins, 7 in. by 9 in. by 2 in. thick, fixed by U-bolts to lugs projecting from the frames and spaced 2 ft. 6 in. apart. The ridge-beams are 18 in. by 9 in. in cross section and rest on the apexes of the frames and on the end walls. The frames were propped until the purlins, ridge-beams,

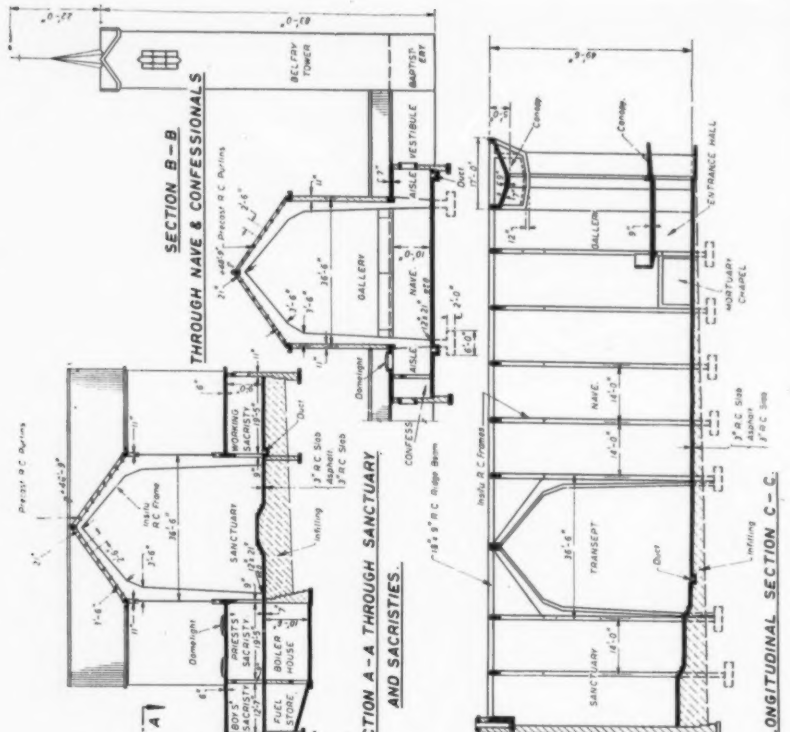

SECTION B–B THROUGH NAVE & CONFESSIONALS
BELFRY TOWER
BAPTISTERY
VESTIBULE
AISLE
GALLERY
NAVE
CONFESS.
Domelight
Precast R.C. Purlins
SECTION A–A THROUGH SANCTUARY AND SACRISTIES.
WORKING SACRISTY
SANCTUARY
PRIESTS' SACRISTY
BOYS' SACRISTY
BOILER HOUSE
FUEL STORE
Insitu R.C. Frame
Infilling
Duct
3" R.C. Slab Asphalt 3" R.C. Slab
LONGITUDINAL SECTION C–C.
Canopy
ENTRANCE HALL
MORTUARY CHAPEL
TRANSEPT
Insitu R.C. Frames
R.C. Ridge Beam
PLAN
HIGH ALTAR
CHURCH
CRECHE AREA
STAIR TO GALLERY

Fig. 2.

and internal walls were in position. The pitched roof over the end of the nave has an intermediate support in the form of a curved beam which spans 27 ft. over the entrance to the vestibules. From 5 ft. below the main ridge-beam a curved slab 8 in. thick is extended at the front of the church to form a canopy about 17 ft. long.

The rear part of the roof contains a sloping reinforced concrete flue which connects the vertical flue with the chimney stack. The sloping flue is supported by a curved beam of varying depth.

The roofs over the aisles are 7 in. thick and span between the columns of the frames and the outer walls. The roof over the confessionals has four circular dome lights of 5 ft. diameter at 14 ft. centres. The slabs are covered with ¾-in. asphalt and foamed-slag concrete laid to a fall.

The sacristies are single-story buildings attached to the church, and under them are a boiler house and a fuel store. The floor of this building is a 7-in. suspended slab with a wood-block finish. The flat roof is a 6-in. slab, in which is formed a gutter.

The belfry tower is 83 ft. high, with cavity walls 20 in. thick. The stairs and landings are of pressed steel, the top floor is a 7-in. reinforced concrete slab, and the roof is a curved slab, 9 in. thick with a rise of 5 ft., spanning in two directions. The roof supports a reinforced concrete spire 12 ft. high, octagonal in cross section, and tapering from 3 ft. at the base to 6 in. at the apex; it is surmounted by a steel cross 10 ft. high. The concrete wall of the spire is 6 in. thick.

Fig. 3.

The architect is Mr. Thomas S. Cordiner, F.R.I.B.A., and the contractors Messrs. John Train & Co., Ltd. The reinforced concrete work was designed by the British Reinforced Concrete Engineering Co., Ltd.

Fédération Internationale de la Précontrainte.

Professor E. Torroja, Director of the Instituto Tecnico de la Construccion y del Cemento, Madrid, has been elected President of the Fédération Internationale de la Précontrainte. M. Freyssinet, the retiring President, has been elected Honorary Vice-president.

The new Committee is as follows. Chairman: Professor E. Torroja (Spain). Deputy-general Vice-president: Monsieur Y. Guyon (France). Vice-presidents: Professor S. S. Davydov (U.S.S.R.), Mr. Ben C. Gerwick (Jr.) (U.S.A.), Dr. G. F. Janssonius (Netherlands), Mr. B. Kelopuu (Finland), Dr. H. Minetti (Federal Germany), Mr. D. H. New (Great Britain). General Secretary and Treasurer: Mr. P. Gooding (Great Britain).

The following countries have been elected to membership of the Fédération: Czechoslovakia, East Germany, Japan, Norway, Poland, Rumania, South Africa, Taiwan, U.S.A., U.S.S.R.

Correspondence.

THE STATUS OF ENGINEERS.

THE Editorial Note in our April number brought messages agreeing with our view that the continual pleading for a higher "status" for engineers was making the profession appear ridiculous. As one of them said, "We are made to look like petulant children clamouring for someone to notice us."

It appears that the essence of the following letter is a repetition of the claim for higher "status", which engineers are urged to achieve by persuading the rest of the people that engineers possess social talent and by fitting themselves to "play a wider part in society". There is nothing whatever to prevent engineers from taking a greater part in the social and political life of the nation if they wish to do so, and we do not know of any other profession that finds it necessary to urge its members in this direction. If the word status is used as a synonym for remuneration, then we are in full accord with the efforts of any man to secure rewards commensurate with the service he renders to the community, and among these services engineering ranks very high.

But we do not think that this propaganda for higher "status" is either dignified or useful. For example, in 1954 claims were reported in the Journal of the Engineers' Guild that "the engineer is a representative of European culture" and "the engineer is predestined and prepared to render particularly valuable co-operation in resolving international political questions". In a lecture reported in the Journal recently a member said that "engineering graduates are being turned out who are only half educated in any real sense. Some are almost illiterate, fitted no doubt to become excellent backroom boys, but little more than that." It is our opinion that the publication of such exaggerated statements can do nothing but harm to the profession, and we shall continue to draw attention to such follies.—EDITOR.

SIR,—

Your own description "jumble of false reasoning" could not be more appropriately applied than to the peculiar fabric of distorted arguments which you have assembled under your Editorial Notes for April 1958. Even if the speaker whom you quote had in fact suggested that we in this country should follow the central European example—and on your own version of his remarks he did no such thing—a suggestion by a single speaker at a branch meeting does not necessarily become immediately the policy of the Engineers' Guild.

It is true that the Guild in pursuit of its objects, which are, "to promote and maintain the unity, public usefulness, honour and interests of the Engineering Profession" has, in the past, considered suggestions for the establishment of a distinctive social title for members of that profession, but has hitherto rejected such suggestions as neither desirable nor practicable. It has also, with some reluctance, reached the conclusion that the misuse of the word "Engineer" by the Press and the public at large is something which has now gone so far as to be past all remedy. Mis-use it nevertheless remains, and at the very least it is a handicap which the engineering profession must carry.

Nobody as far as I am aware, either in the Guild or outside it, has ever put forward the extraordinary theory which you attribute to "some members of the Guild" that improved social status would immediately and automatically generate superior professional and technical effort by engineers, but you are surely not unaware that the need to attract into the profession a larger proportion of the country's best brains than finds its way there at present, is a matter which is exercising not only the Engineers' Guild, but the engineering Institutions themselves, and even the Government, as the 1956 white paper on Scientific and Engineering Manpower gives evidence.

Human nature being what it is, it is inevitable that, in a free society, the motives which may draw a man to one sphere of activity rather than another, are bound to be somewhat mixed, and the level of material reward to be gained, and the degree of social esteem to be achieved

must certainly play their part in influencing the decisions of all save the dedicated very few. Even in those societies where the minds and bodies of the citizens are allegedly at the complete disposal of the central authority, the appeal to the same motives is evident, and it is hard not to see some connection between the immense technological strides being made in the Soviet Union and the position of high privilege enjoyed by the practitioners of technology in that country.

Turning to your analogies from other professions, if your contact with feeling in the Royal Air Force is no closer than it evidently is in the case of the Engineering Profession, then your suggestion that the application of flying titles to people who never leave the ground does not cause any heartburning, is one which must be treated with a certain amount of reserve. In similar circumstances in the Army, the coining of the term "base wallah" might suggest a certain degree of emotion on such a subject. Doctors of Divinity are very generally in Holy Orders, and are therefore accorded the title—Rev., Ven., etc.—which normally goes with that vocation. A large proportion of medical practitioners are in fact only qualified as Bachelors of Medicine, so any objection on their part to the use of the term "Doctor" by somebody who had in fact earned the qualification, would be somewhat out of place. Indeed, the whole relevance of the fact that apparently nobody objects to the use of the title "Doctor" by anybody on whom the qualification has been bestowed, to the application of the description "Engineer" to people who are in fact not Engineers, is not readily apparent.

The Engineers' Guild does not suffer from what you call a craving for public esteem, but we feel strongly that in a world in which practically everything that makes modern society physically possible, is attributable to the activities of Engineers, the Engineering Profession has an important and valuable part to play in society outside the immediate scope of the technologies which it practices. Indeed, your comments on this and on previous occasions lead to the impression that in your opinion the Engineer should confine himself to his workshop, his drawing board, his constructional site or his laboratory, should be in fact a backroom boy, and leave the higher direction of society to presumably superior beings. The Engineers' Guild does not share this narrow concept of the functions of the Engineering Profession, and therefore we do what we can to persuade the members of that profession to fit themselves to play a wider part in society than most of them have been accustomed to do in the past, and to persuade society of the value of the pool of social talent which is at their disposal in persons who have been subjected to the discipline of an engineering training.

We do not ignore the importance of high standards of teaching and training, but these are matters which are primarily the concern of the Engineering Institutions. Just as the latter are precluded by their Charters from concerning themselves with many of the activities which occupy the Engineers' Guild, so the Engineers' Guild as a matter of courtesy would not dream of disturbing the friendly relations it at present enjoys with the Institutions by attempting as it were to show the latter how to do their job. Some of us indeed are, or have been, members of the Councils of both the Engineers' Guild and our appropriate Engineering Institution, which suggests that it is possible for the same individual to pursue both aspects of the improvement of the Engineering Profession with equal zest.

The ideas which you have wrongly attributed to the Guild *are* nonsense, and *would* be unworthy of a great profession if anybody had seriously set out to propagate them on its behalf.

Finally, Mr. Smith would probably not be a better architect if he were to be known as Mr. Architect Smith, but for reasons which apparently seemed to him sufficient, he went to considerable trouble a number of years ago to see to it that if Mr. Builder Jones were surreptitiously to call himself Mr. Architect Jones, he would be open to legal procedures. It would seem therefore that some other professions have this same unregenerate attachment to name and titles which you find so unworthy in the Engineers' Guild.

Recognising the limits of what is practicable, and not in fact overlooking the antiquity of the A.S.L.E.F., and the A.E.U., to which you draw attention, the Engineers' Guild urges all those members of the Engineering Profession entitled to do so, to use, in appropriate circumstances, the description "Chartered"

which is protected for their use by the legislation of the country. For this reason, Sir, I sign myself

Yours faithfully,
J. H. JELLET,
Chartered Civil Engineer, M.I.C.E.,
Chairman of the General Council,
Engineers' Guild.

[In the article commented on in the foregoing it was stated that the lecturer claimed that the use of the word Ingenieur before the names of engineers in Germany was the cause of their high social status. This was inaccurate. His claim was that the use of this title "reflected the high social status given to the engineer in Germany, a status that had assisted and encouraged the remarkable technological progress in that country, particularly in post-war years".]

Demolition of Concrete by Explosives.

INVESTIGATIONS by the Stanford Research Institute in California, on behalf of the U.S. Army, to determine the quantity of TNT required for the demolition of concrete have been recently completed. It was found that the quantity required to break a wall or pier can be determined from the formula $P = R^3KC$, in which P is the weight of TNT in pounds, R is the thickness of the wall in feet, K is a factor as given in *Table* I, and C is a tamping factor. When the charge is not tamped with wet sand or earth, $C = 1{\cdot}0$; when it is tamped, $C = 0{\cdot}6$. The charge should be placed at a minimum height of R above the base of the wall; the smallest recommended charges are 3 lb. for unreinforced walls and 5 lb. for reinforced walls.

The formula applies to the demolition of beams if the charge is tamped with sandbags at least 10 in. thick on every side. These charges are stated to shatter the concrete sufficiently to allow it to be removed easily. They will not break the reinforcement.

R	Unreinforced	Reinforced
Less than 3 ft.	0·95	1·66
3 ft. to 5 ft.	0·72	1·28
5 ft. to 7 ft.	0·54	1·00
7 ft. and over	0·56	0·89

Prestressed Stone.

THE following is an abstract of a paper describing Polish investigations of the prestressing of natural stone presented by Mr. J. Pietrzykowski to the third congress of the Fédération Internationale de la Précontrainte held in Berlin in May last.

The stone tested was found to be very suitable for prestressing. The compressive strength, effects of frost, and Young's modulus for some types of granite and sandstone were studied on cubes of about 3 in. sides. The load-carrying capacity of prestressed stone beams about 16 ft. 6 in. long and made of blocks measuring about 9½ in. by 16 in. by 40 in. was investigated; the actual values exceeded the predicted values by 5 to 15 per cent.

The most suitable cross section was found to be that shown in *Fig.* 1, the

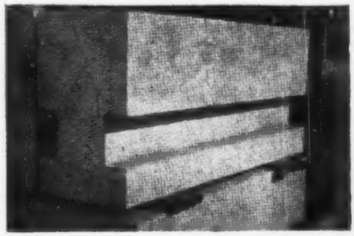


Fig. 1.

joint between the blocks comprising cement mortar about ¼ in. thick which had been subjected to initial pressure and with the prestressing steel in the rebates in the sides. The cost of quarrying the stone, making the elements, and building stone structures with straight prestressing cables was found to be economical.

The principal advantages of stone as a structural material are its durability and compressive strength. Fine-grained granites of Silesia have a strength of about 28,500 lb. per square inch and sandstones with siliceous binders about 14,000 lb. per square inch. A further advantage is that the rheological processes, which are the cause of considerable losses of prestress in concrete structures, are almost completed in natural stone. The principal disadvantages of stone are the possibility of hidden cracks and structural defects, and the lack of homogeneity of some stones.

Automatic Control at a Ready-mixed Concrete Plant.

A PLANT for the production of ready-mixed concrete at Mitcham, Surrey, which was recently opened by the Minister of Works, incorporates a fully-automatic system of control and will be operated by two men. It is thought to be the most up-to-date plant of its kind in the world. *Fig.* 1 is a general view, and a longitudinal section is shown diagrammatically in *Fig.* 2.

Handling of Materials.

Sand and gravel are transported to the plant in tipping trucks, and delivered into four hoppers (*Fig.* 3) each of which has a capacity of 12 cu. yd. and has a radial gate operated through hydraulic thrusters remotely controlled from a penthouse at the top of the mixing tower. The aggregates are removed from the hoppers by a horizontal troughed-belt conveyor 30 in. wide, and transferred to an inclined trough-belt conveyor, 30 in. wide and 304 ft. long, which delivers each size of material, through a swivelling chute, to the appropriate compartment of the storage bin at the top of the tower. The conveyor has a capacity of 150 tons per hour.

Loose cement is delivered in tipping lorries and is discharged into two receiving hoppers, each with a capacity of 15 tons, which are housed in a brick structure sufficiently large to accommodate the vehicle. A rubber sheet is placed over the rear of the truck, and the pressure in the receiving hopper is less than that of the atmosphere in order to prevent the escape of cement while it is being discharged.

An air-flow system in the base of each hopper conveys the cement to an activator, from which it is delivered in aerated batches to a storage silo (*Fig.* 4) with a

Fig. 1.—General View of Plant.

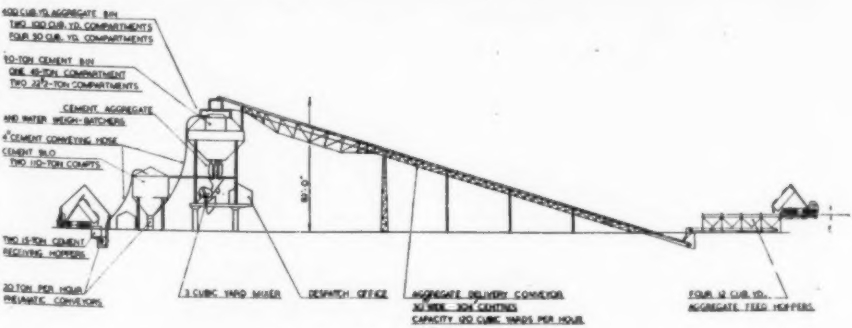


Fig. 2.—Longitudinal Section.

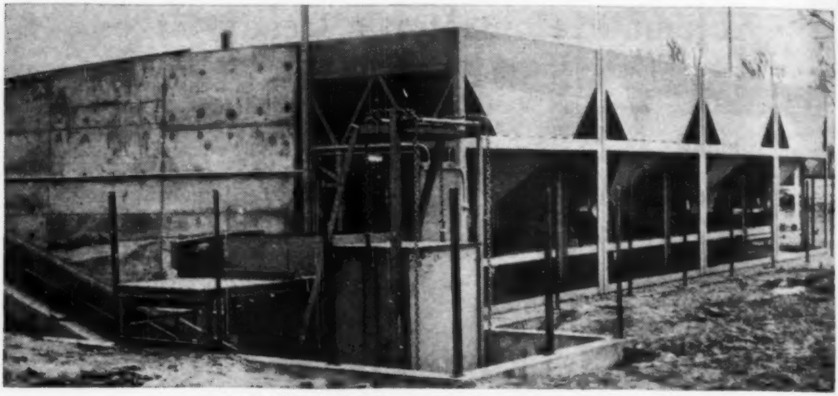


Fig. 3.—Storage Bins for Aggregates.

capacity of 220 tons, or to a storage bin at the top of the mixing tower. The silo has two compartments, from which the cement is reclaimed and elevated to the bin in the mixing tower by means of two more activators. The installation has a combined rated capacity of 40 tons per hour. If necessary cement can be delivered to the storage silo from pressurised vehicles.

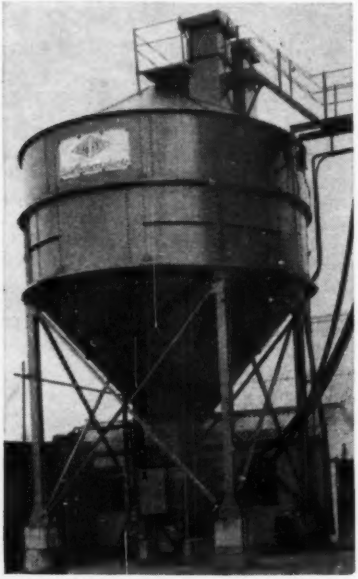


Fig. 4.—Cement Storage Silo.

Water is stored in a tank with a capacity of 7000 gallons, and is delivered to the water-weigher by a 2-in. centrifugal pump.

The Mixing Tower.

The aggregates and cement are delivered to a storage bin at the top of the mixing tower. The bin is 35 ft. in diameter and can accommodate 400 cu. yd. of aggregate in six compartments arranged radially around three central cement compartments; these usually contain ordinary, rapid-hardening, and sulphate-

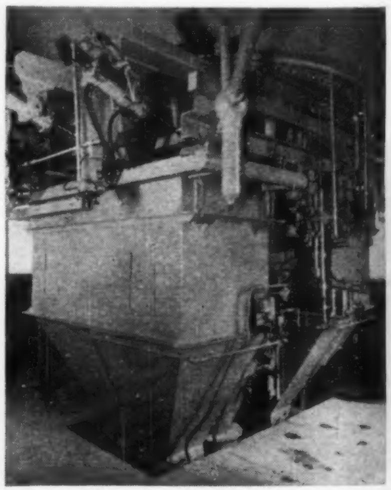


Fig. 5.—The Weighers.

resisting cements. The cement compartments are of 90 tons capacity; they have roof plates and ventilators and are sealed to provide protection from the weather. The aggregates are discharged from the bin to the weighers (*Fig.* 5) through undercut radial gates operated by pneumatic rams. The cement passes to the weigher through a rotary valve operated by pneumatic rams. Air jets are provided at the bottom of the cement compartment to ensure an even flow.

The weighing of the materials is controlled by a fully-automatic punched-card process. Provision is made for emergency manual operation. Three weighing hoppers are provided; the central hopper can weigh up to 2800 lb. of cement, and a small vibrator is incorporated to ensure complete discharge. The other two hoppers, each with a capacity of 8000 lb., weigh the aggregates cumulatively. The water-weigher has a capacity of 1500 lb.

The proportioning is controlled by a man in an office on the lower deck of the tower, who can at all times observe the discharge of the mixer (*Fig.* 6), and who inserts a punched card into an electronic "card-reader", which controls an automatic proportioning device (*Fig.* 7); this selects the required materials and weighs the batch.

A system of interlocks ensures that only the required quantities of materials can be delivered to the mixer. To allow for variations in the moisture content of the sand, a device is incorporated by which the weight of sand is increased and the weight of water correspondingly reduced by the necessary amount. This can be adjusted by means of a rotary switch on the panel, which is calibrated to allow for a water content in the sand of 2 to 12 per cent.

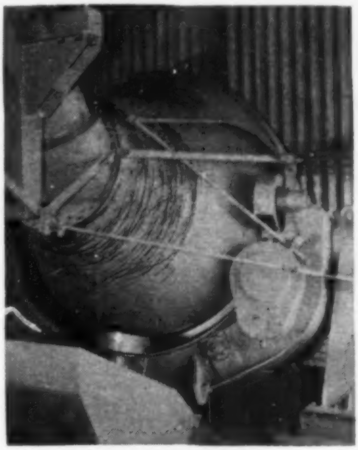


Fig. 6.—The Mixer.

A tilting-drum mixer (*Fig.* 6) with a rated capacity of 3 cu. yd. of mixed concrete is used, and provision is made for the installation of a second mixer. The mixer is electrically driven, and is tilted by two pneumatic rams; the degree of tilt is controlled from the despatch office. A system of interlocks prevents the discharge of a batch from the weighing hoppers until the mixer has discharged the previous batch. When the mixer is in the correct position for charging, the hopper gates open automatically; they

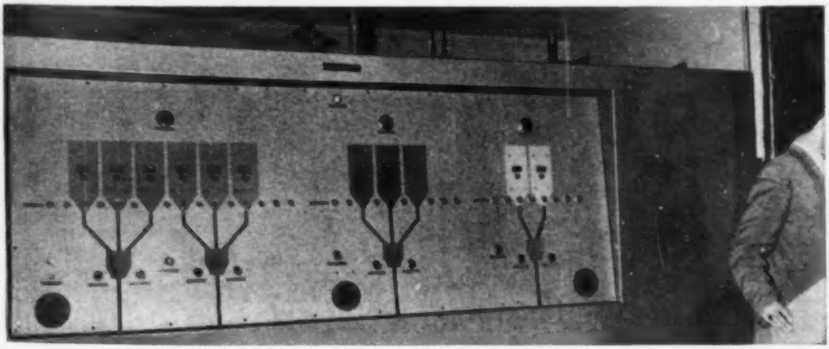


Fig. 7.—The Proportioning Panel.

can be adjusted to open in sequence. Pre-set timing devices control the automatic closure of the gates, and an indicator shows when the hoppers are completely emptied. The required mixing time is set on an adjustable timing device, and the mixer cannot be discharged until after this period has elapsed.

The concrete is discharged into a hopper with an outlet to the agitating vehicle which transports the concrete.

All the British Standard tests of aggregates and concrete are made in a laboratory at the plant. The equipment includes a compression-testing machine with a capacity of 200 tons and compaction-factor apparatus. The laboratory staff is responsible for the selection of the materials and the proportions and the accurate punching of the cards.

The plant, which cost about £150,000, was supplied and erected by Blaw-Knox, Ltd. It is owned and operated by Messrs. Hall & Co., Ltd.

Concrete Wine Vats in Portugal.

THE existing storage facilities were insufficient to accommodate the unusually large output of Portuguese wine in 1956, and fifty additional vats were constructed in ninety days. The vats are hemispherical in shape, and each contains 73,600 gallons of wine (*Fig.* 1). To construct each tank, a neoprene-nylon balloon was fastened to a foundation ring and inflated. Reinforcement was then placed in position and a layer of concrete 2 in. thick was sprayed on. When the concrete had hardened the balloon was deflated and re-used. Similar vats will be constructed to replace existing accommodation that may become unserviceable. The agents in Great Britain for Neoprene synthetic rubber are Durham Raw Materials, Ltd.

Fig. 1.

Atomic Power Station in Somerset.

A CONTRACT for the construction of the world's largest atomic power station at Hinkley Point, Somerset, is being carried out by the atomic power group of the English Electric Co., Ltd., Babcock & Wilcox, Ltd., and Taylor Woodrow, Ltd. The station will occupy 44 acres, and is designed for an output of 500,000 kW. The power will be generated at a pressure of 13·8 kV., and transformed to 275 kV. to supply the national high-voltage grid. There will be two reactor buildings with their ancillary structures, a turbine house (to which will be attached a workshop building), a control building, and administration and welfare buildings (*Fig.* 1). Three boilers and their related carbon-dioxide-blower house will be grouped on either side of each of the two reactor buildings, with two used-fuel storage ponds behind them. The administration, welfare, and canteen buildings will be built around three sides of a rectangular garden, the front of which will be faced by the workshop attached to one end of the turbine house.

The heavy loads of the reactors will be transmitted to sub-strata of limestone and shale through separate reinforced concrete foundations. The floor of the basement of the turbine house will be 30 ft. below finished ground level, and the roof will be 61 ft. above the basement floor.

The reactors will be of the gas-cooled graphite-moderated type, producing nearly 1000 megawatts of heat, and will be connected to six axial-flow gas circulators and six boilers. The fuel will be natural uranium. The reactors consist of a graphite-moderated core with a lattice of uranium fuel contained in a spherical pressure vessel 67 ft. diameter and made of steel 3 in. thick. Radiation from the reactor will be kept very low by thermal and reinforced concrete biological shields. The fuel will be charged into the top of the reactor and will be done on load to permit a continuous fuelling cycle and to enable the elements to be moved about to obtain the maximum use of the fuel. A gas-sampling system will ensure early detection of any increase in radioactivity in the coolant gas, and allow means to be taken to avoid contamination of the reactor. Control gear and rods will be removable for maintenance while the reactor is working at full power.

The station will have six main 93·5-megawatt turbo-alternator sets, each 104 ft. long. The sets will be 3000-r.p.m. impulse-reaction machines with a high-pressure cylinder and two double-flow low-pressure cylinders with solid-coupled shafts, each of which will exhaust into a separate condenser. There will also be three variable-speed turbo-alternators to supply power for the gas circulators.

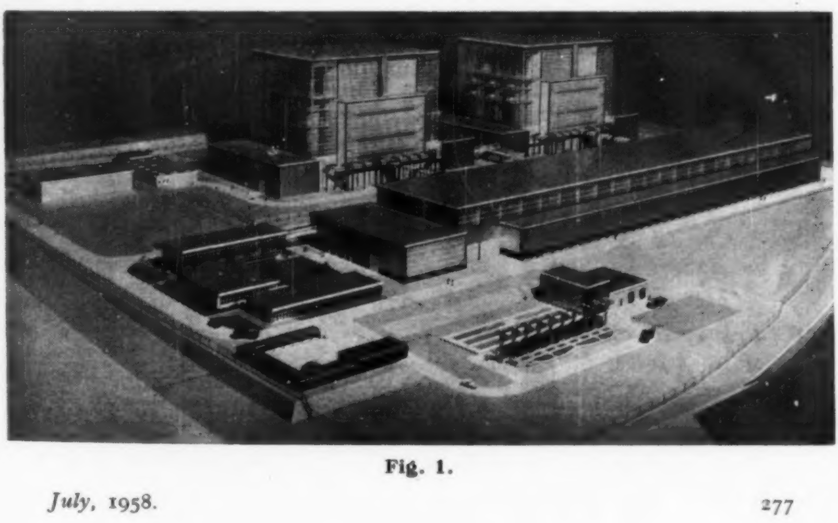
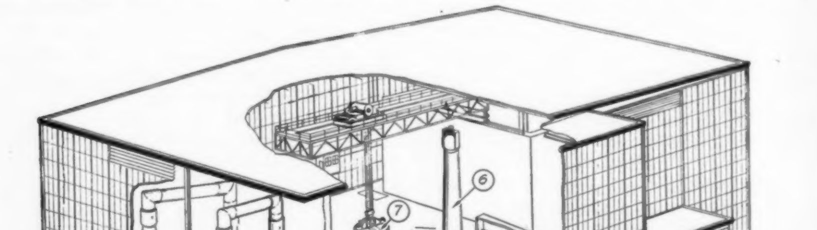


Fig. 1.

Dump condensers will be provided for commissioning purposes and the removal of reactor shut-down heat. Cooling water, which will not be subjected to irradiation, will be drawn from and returned to the sea through centrifugal pumps. A decontamination area will have facilities for storing and decontaminating irradiated components. Provision will also be made for storage of new and spent fuel elements. Changing-rooms will be provided for people entering active areas. The operation of the station will require a staff of about 300.

The Reactors and Pressure Vessels.

The 24-sided core of each reactor will be made of graphite blocks which will contain, and act as a moderator for, the fuel elements and form a reflecting envelope surrounding the core (*Fig.* 2). The structure will be held in position by bands around the periphery. It will be contained in a spherical steel pressure vessel and supported on ball-bearings to reduce friction caused by differences in thermal expansion between the metal supporting-plates and the graphite. The fuel will be loaded at the top through charging-pans which will protect the top face and position the charging machinery. Gas-seals will control the flow of coolant gas between the graphite structure and the pressure vessel. The core will be pierced by a lattice of vertical channels through which the fuel elements will be lowered into the moderating graphite blocks, and holes for control rods, flux scanning, and irradiation test specimens will be formed from chamfers at the four corners of



1.—Pressure vessel. 2.—Graphite core. 3.—Drive for control rods. 4.—Cone supporting the grid. 5.—Biological shield. 6.—Machine for preparing holes. 7.—Fuelling machine. 8.—Maintenance room. 9.—Axial-flow gas blower. 10.—Boiler. 11.—High-pressure steam drum. 12.—Low-pressure steam drum. 13.—CO_2 inlet duct. 14.—CO_2 outlet duct. 15.—Exhaust fan. 16.—Switchgear. 17.—Precipitator room.

Fig. 2.

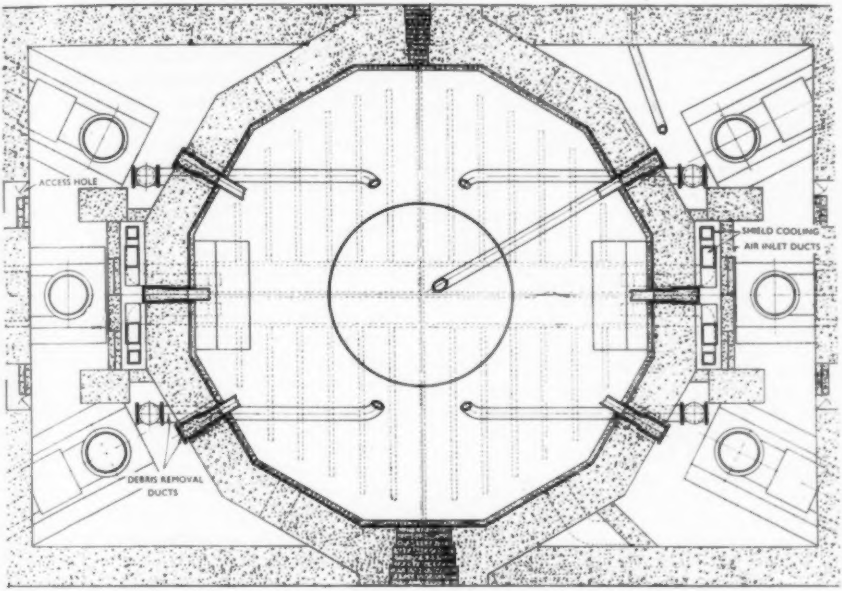


Fig. 3.

adjacent columns. In the event of the inadvertent dropping of a fuel element or control rod, the shock will be absorbed by special devices. The temperature of the graphite will be measured by thermocouples, and allowance is to be made for the expansion of bricks due to changes of temperature and radiation without affecting the geometry of the lattice.

The spherical pressure vessel to contain the graphite core will be constructed of preformed mild-steel plates which will be welded on the site. The vessel will be supported on an external cylindrical skirt welded to it, the bottom of which will rest on structural steel members above the concrete foundation. The graphite structure will be supported on a grid, the weight being transmitted to the external skirt by an internal skirt supporting the grid. Tubes welded to nozzles inside the vessel will project through the biological shield to enable such operations as the charging and discharging of fuel and the insertion of control rods to be performed. Thermal insulation will consist of internal lagging in the upper half of the vessel and external lagging on the lower part.

The Shield.

The pressure vessel will be wholly contained within a reinforced concrete structure, known as the " biological shield ", to protect the staff from the radiation of the reactor (*Fig.* 3). This will be lined with a thermal shield. The two shields will be separated by passages of varying width for cooling air, so that the thermal shield, with the cooling system, will protect the biological shield from excessive temperatures and temperature gradients. The continuous flow of cooling air will be discharged to atmosphere through filters designed to stop the passage of any irradiated particles.

The main shields will be duodecagonal in shape, with reinforced concrete walls 7 ft. thick and more than 90 ft. high. The internal diameter will be about 75 ft. Reinforced concrete pile-caps, 11 ft. thick, will surmount the shields. Secondary shields in the form of 140 ft. by 100 ft. reinforced concrete rectangles will attenuate radiation which may escape through the openings in the main shields ; these will be 6 ft. 6 in. thick.

Boilers and Turbo-Alternators.

The six boilers associated with each reactor will generate more than 5,500,000 lb. of steam per hour, two-thirds of which will be high-pressure and one-third lower-pressure steam. The main vertical shells of the boilers will be 90 ft. high and 21 ft. 6 in. diameter, with a maximum wall thickness of 2⅜ in., and will be mounted on skirt-supports resting on a concrete base.

The six main 300-r.p.m., hydrogen-cooled, turbo-alternator sets will each have a continuous maximum rating of 93·5 megawatts at a generated voltage of 13·8 kV. and 0·85 power factor. Three 33-megawatt variable-speed turbo-alternators, one of which will act as a standby, will supply power for the gas blowers and will be of sufficient capacity to allow for increased output from the reactors.

The main turbines will be of the horizontal impulse-reaction type with a high-pressure cylinder and two low-pressure cylinders. They will operate on the mixed-pressure cycle, each low-pressure cylinder exhausting to its own condenser, and will be equipped with a feed-heating system consisting of a drain-cooler and a feed-heater.

The hydrogen-cooled alternators will work at a rated pressure of 15 lb. per square inch gauge of hydrogen but are designed for a maximum operating pressure of 30 lb. per square inch. The turbines of the variable-frequency sets will be of the horizontal two-cylinder impulse-reaction type, operating at variable speed.

The station will require about 35,000,000 gall. of water per hour for cooling purposes. Sea-water, circulated by six main centrifugal pumps, will be taken in through a structure precast in a dry dock on shore and floated out to a position 2200 ft. offshore. After passing through two tunnels, each 11 ft. in diameter, to the pump-house on shore and circulated through the station, the water will be passed to an outlet channel on the foreshore, remote from the intake, through nearly 2½ miles of culverts, and discharged to the sea. Access to the intake from the shore will be provided by a tunnel of 7 ft. diameter which will join the intake at the pump-house.

Other civil engineering works will include a plain concrete sea-wall about 3000 ft. long and a wharf at Combwich six miles from Hinkley Point. The construction of the station and its ancillary works will require about 300,000 cu. yd. of reinforced concrete, 12,000 tons of reinforcement, 400,000 tons of aggregates, and 8500 tons of structural steel.

Concrete-Encased Steel Stanchions.

ECCENTRIC-LOADING tests have been made at the Building Research Station on concrete-encased stanchions to obtain further information on the strengthening and stiffening effects of concrete encasement provided for fire protection of the stanchions of steel-framed buildings. The results supplement those of earlier tests in which similar stanchions were loaded axially.

The axial compression tests showed that when a stanchion is encased with a cover of concrete sufficient for a fire resistance of two hours the encasement can increase the ultimate axial strength of the stanchion considerably. The ultimate axial strength of short encased stanchions can be estimated by the method established for short reinforced concrete columns, and there is a reduction of the ultimate strength as the slenderness of the stanchion is increased. In laboratory tests this reduction, particularly for slender stanchions, was found to be smaller than that given by the design formula of British Standard Code of Practice No. 113 : 1948 and B.S. 449 : 1948. When encased stanchions of slenderness ratios up to 120 were loaded with small initial eccentricities, their ultimate strengths were found to be in good agreement with values calculated by the elastic theory (assuming that failure in compression occurred at the outer face of the concrete at a stress 33⅓ per cent. greater than the ultimate axial compressive strength of the concrete and ignoring increasing curvature of the stanchion under load). A simple factor could therefore be applied to the ultimate axial strength to provide a fair estimate of the ultimate eccentric load under the test conditions.—(From "Building Research 1956". H.M. Stationery Office. Price 5*s*. 6*d*.)

Symposium on the Observation of Structures.

A REPORT of the Symposium on the Observation of Structures, held in Lisbon in the year 1955, has now been published by the Laboratório Nacional de Engenharia Civil, at Lisbon, in two volumes. The following papers are given, some in the French language and some in English.

Austria.—Crack measurements in reinforced concrete structures, by Stefan Soretz.

Belgium.—Auscultation of the Ardennes bridge at Namur, by E. Dehan and J. Thonnard. New apparatus for structure auscultation, by E. Dehan and E. Demol. Permeability and capillarity, by M. van Laethem. Results of the auscultation of two metallic road bridges and one metallic railway bridge, by F. Hebrant, H. Louis, and A. Soete. Results of the uninterrupted auscultation since 1946 of a prestressed concrete bridge on the Meuse at Sclayn (Belgium), by H. Louis and E. Dehan.

Canada.—Photogrammetric measurements of deformations of structures, by C. Moser and W. R. Schriever.

England.—Recent developments in the design and use of strain gauges for the full-scale testing of structures, by H. D. Morgan and K. R. McLachlan. Recent investigations on building structures, by S. R. Sparkes and J. C. Chapman. Special techniques developed at the Road Research Laboratory for testing roads and other structures, by A. C. Whiffin. The measuring devices installed in the Mullardoch and Cluanie dams, Scotland, by C. M. Roberts. Strain measurements on Telford's suspension bridge at Conway, Wales, by J. B. B. Owen. Testing techniques used by the Building Research Station to measure existing stresses, by N. Davey. The measurement of pore-water pressure and its application to some engineering soil problems, by L. F. Cooling.

France.—A special case of auscultation by surveying methods, by M. Mermin. Auscultation of dams and dam sites by surveying methods, by J. Nevière and P. Blanchet. Auscultation of dams by means of pendulums, clinometers, and acoustic strain gauges, by G. Willm. Compressibility of foundation rocks under dams, by J. Bellier and W. Ter Minassian. Determination of the bearing strength of airfield runways, by R. Lorin. Dynamometers for prestressed parallel wire cables, by E. Petit and J. Bellier. Experimental proof of the unchanging reliability of vibrating strings, by J. Bellier and L. Sabbat. Measurement of dynamic effects by means of vibrating strings, by G. Jacubowitz and J. Bellier. Non-destructive methods of stress analysis by means of photoelastic varnishes, by F. Zandman. Some remarks on the making of torquemeters, by R. Comolet. Tests to failure of a coated girder railway bridge, by A. Lazard. Thermic auscultation of dams by means of thermocouples, by J. Chapelle. Two new models of recording micrometer dial gauges, by G. Dawance.

Germany.—Determination of damage in reinforced concrete constructions with ultrasonics, by K. Wesche. Results obtained in testing structures by means of dynamic measurement, by H. W. Koch.

Israel.—Interpretation of strength distribution as a factor in quality control of concrete, by Rahel Shalon and Rudolph C. Reinitz.

Italy.—Deformation control in reinforced concrete structures by means of extensometers immersed in the mass, by J. Bossi and G. Moravia. Application of the photoelastic method to the measurement of deformations in elastic-plastic structures, by Giuseppe Grandori.

Portugal.—Measurement of strains in concrete dams, by J. Laginha Serafim. Measurement of the absolute displacements of large dams, by O. Vieira Rodrigues. The observation of road and railway embankments and slopes, by Ulpio Nascimento and Abel Simões. Observations of dam displacements by means of pendulums, by A. F. da Silveira. Observation of dams: methods and apparatus used in Portugal, by Manuel Rocha, J. Laginha Serafim, and A. F. da Silveira. Infiltrations through some Portuguese dams and their foundations, by Armando da Palma Carlos, Luís Júdice Folque and Adelino Sintra Nunes. Joint movements in dams, by A. F. da Silveira. Settlements in the Benagazil canal aqueduct of Valo do Sado irrigation scheme, by Armando da Palma Carlos, Luís Júdice Folque, and Adelino Sintra Nunes. Observation of some reinforced concrete structures, by Manuel Rocha, J. Ferry Borges and José Marécos. The evaluation of the safety of old metallic bridges, by

J. Ferry Borges, José Marécos, and L. da Cunha Ferraz. The use of precise geometric levelling for the observation of structures, by A. Ferrer de Lemos. Safety control of arch dams, by A. Carvalho Xerez. Tests of a pressure gallery lining, by Luís M. Lobo.

Sweden.—Stresses and deformations in the Krokströmmen arch dam, by Erling Reinius. Temperature measurements with thermistors in concrete, by Knud E. C. Nielsen and Ingvar Schill.

Switzerland.—Sensitivity of resistance strain gauges to transverse deformations, by A. U. Huggenberger. The observation of concrete structures, by A. U. Huggenberger. Measuring instrument for forces, by Ulrich Schlumpf.

U.S.A.—Determination of stresses in an arch dam from observed strains, by A. Warren Simonds. Effect of longitudinal joints on the stresses at the base of an arch dam, by Jerome M. Raphael. Studies of torsion in suspension bridge stiffening trusses, by Sidney Shore.

Yugoslavia.—Experimental determination of the modulus of elasticity of rocks, by Branislav Kujundzic.

A Coloured Ceramic Aggregate.

A METHOD of applying a white or brightly-coloured surface to concrete by means of ordinary spraying equipment has recently been developed. The concrete to be treated is covered with a bituminous or plastic emulsion, and a granular ceramic aggregate of the required colour is sprayed on. If desired, the surface can then be coated with clear lacquer.

The cost of the aggregate is stated to be about 1*s*. per square yard ; the total cost of the materials is about 4*s*. 6*d*. per square yard if plastic emulsion is used or 3*s*. 6*d*. per square yard using a bituminous emulsion. Coloured pebbles can also be obtained. Red, blue, yellow, and white aggregates are available. The material is supplied by Dohm, Ltd.

FIFTY YEARS AGO.

FROM "CONCRETE AND CONSTRUCTIONAL ENGINEERING", JULY–AUGUST, 1908.*

[This photograph was taken on March 27, 1908, and shows the main arches of a bridge at Philadelphia, U.S.A. The arch has a span of 233 ft. and a rise of 70 ft. ; plain concrete in the proportions of 1 : 2 : 5 was used.]

* "Concrete and Constructional Engineering" was published in alternate months until September, 1909.

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